CONSIDERATIONS ON THE SLIP DEMAND OF SHEAR CONNECTORS IN COMPOSITE STEEL-CONCRETE BEAMS WITH SOLID SLABS

A. Zona [1] and G. Ranzi [2, *]
[1] School of Architecture and Design,
University of Camerino, Ascoli Piceno, Italy.
[2] School of Civil Engineering,
The University of Sydney, Sydney, Australia. *Email: gianluca.ranzi@sydney.edu.au

ABSTRACT

The objective of this study is to provide insight into the expected slip demand in composite steel-concrete beams through numerical simulations. A wide parametric analysis is carried out evaluating the partial interaction performance of simply-supported beams designed considering a variety of floors, i.e. span length, slab thickness, shear connection strength, dead load to live load ratio and slab concrete strength. For each of these beams, the slip demand required to achieve the expected design capacity is evaluated. In this process, key parameters influencing the slip requirements are identified. These also include the construction sequence (propped or unpropped) and the shear connection distribution (uniform or non-uniform with different layouts).

KEYWORDS

Composite steel-concrete beams, nonlinear analysis, partial interaction, shear connection, slip demand, Eurocodes.

INTRODUCTION

The design of composite steel-concrete beams requires specific attention to the shear connection, fundamental for achieving an effective and efficient collaboration between the steel and reinforced concrete components. The design should include not only the evaluation of the required strength of the shear connection system but also its slip capacity compared to the slip demand at the ultimate limit state, e.g. (Oehlers et al, 2001; Oehlers and Sved, 1995). Many experimental and numerical studies have been performed on the behaviour of various connections systems aimed at assessing their capacity at the ultimate limit state. On the other hand, less attention has been devoted to establishing the expected slip demand at the ultimate limit state, despite the numerous research work that in the past decades involved the nonlinear analysis up to collapse of composite beams. In this context, the objective of this work is to review some results recently published in (Zona and Ranzi, 2014) involving a study on the expected slip demand in composite steel-concrete beams as used in building applications through numerical simulations. A wide parametric study is carried out evaluating the partial interaction performance of simply-supported beams designed considering a wide range of floor scenarios, i.e. varying span length, slab thickness, shear connection strength, dead load to live load ratio and slab concrete strength. For each of these beams, the slip demand required to achieve the expected design capacity is evaluated. In this process, key parameters influencing the slip requirements are identified. These also include the construction sequence (propped or unpropped) and the shear connection distribution (uniform or non-uniform with different layouts). The results are expected to provide an overview of the key parameters influencing the slip deformations of composite steel-concrete beams, useful in future modifications and refinements of current design recommendations regarding the required ductility of the shear connection, currently implemented as limitations of the minimum degree of shear connection.

EVALUATION OF THE SLIP OF THE SHEAR CONNECTION

Nonlinear structural model

The nonlinear behaviour up to collapse of steel-concrete composite beams is predicted in the parametric analysis of this study by using a material-nonlinear beam model with partial interaction where two Timoshenko beams in small deformation, one for the steel beam and one for the reinforced concrete slab, are coupled by an interface model with a continuously distributed bond, allowing interlayer slip and enforcing equal vertical deflection and rotation between the steel and concrete components (Zona and Ranzi, 2011). Numerical solutions are obtained through assemblies of a 16 degree of freedom (DOF) finite element, previously validated against a wide range of experimental tests, shown to provide accurate predictions of the experimental behaviour up to collapse and good numerical efficiency in terms of convergence for both local and global response quantities (Zona and Ranzi, 2011). The adopted nonlinear constitutive models are described in details in reference (Zona and Ranzi, 2011). Considering its importance for the purpose of this study, the material model adopted for the shear connection constitutive model is briefly outlined. Its expression is based on the Ollgaard et al. (1971) relationship, which consists of an empirical equation providing good approximation of experimentally measured load-slip curves regardless of the failure mode, i.e. concrete failure in the region of the shear connectors, failure of steel studs, or combined failure of the two. This can be expressed as follows:

$$p_s = \mathrm{sgn}(\delta) \cdot p_{s,\max} \cdot \left(1 - e^{-\beta \cdot |\delta|}\right)^{\alpha} \tag{1}$$

where δ is the slip between the two components of the composite beam, p_s is the value of the shear force corresponding to δ, $p_{s,\max}$ is the connection shear strength (determined as the smaller value between the force causing failure in the concrete slab and the force causing failure of the connector), α and β are parameters controlling the stiffness (slope of the curve) for small and intermediate (of the order of β^{-1}) values of the slip, respectively, and sgn(…) is the sign function. The curve in Equation (1) tends asymptotically to the rigid-plastic model for α approaching zero. Since the stiffness at the origin is infinite when $\alpha < 1$, the exponential law is substituted with a linear segment (with stiffness k_a) from the origin to an assigned slip $\delta_a = 0.01$ mm (with $\delta_a << \beta^{-1}$) to avoid numerical problems. The shear connection is supposed to have infinite ductility to evaluate the maximum expected slip demand. The shape of the constitutive model for typical values (Zona and Ranzi, 2011), i.e. $\alpha = 0.588$ and $\beta = 1$ mm^{-1}, is depicted in Figure 1 (continuous line) together with other variations of the values for α and β considered in this study (dotted and dashed lines).

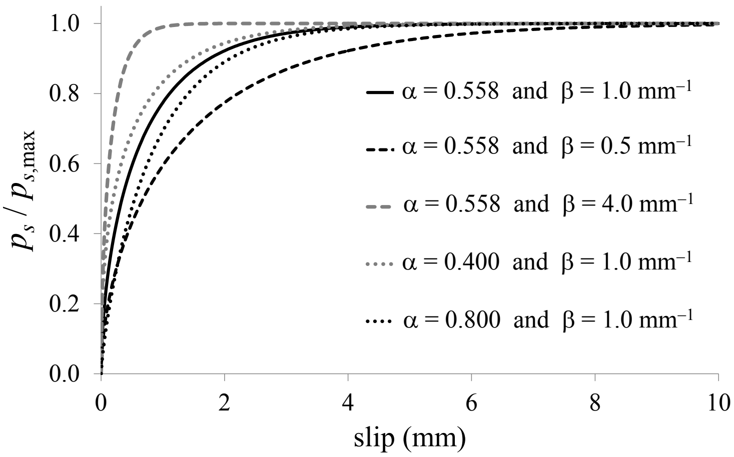


Figure 1 Shear connection constitutive law with adopted parameters.

Safety format for nonlinear analysis

In this study, given that the beams considered in the parametric analyses are designed according to Eurocode 4 Part 1 (Eurocode 4, 2004), the nonlinear analyses are performed according to the indications of the Eurocodes for the nonlinear analysis of composite steel-concrete structures, i.e. paragraph 5.4.3 of Eurocode 4 Part 1 and relevant parts there quoted, as illustrated in (Zona et al., 2010). According to these recommendations, the structural model must include all relevant failure modes and the nonlinear behaviour of the materials (concrete, steel of reinforcements, constructional steel, shear connectors) must be described by the mean values of their constitutive properties. The model should include the behaviour of the shear connection, necessary to realistically evaluate the shear connection slip demand, while it does not have to rely on concrete tensile strength as a primary load resisting mechanism. The nonlinear analysis is performed with loads increased from their serviceability values by incremental steps, so that the value of $\gamma_G G_k$ and $\gamma_Q Q_k$ are reached in the same step, where G_k and Q_k denote the characteristic values of permanent and variable actions, respectively, and γ_G and γ_Q are the partial factors for permanent and variable actions, respectively (in buildings γ_G =1.35 and γ_Q = 1.50 in locations where loads are unfavourable while γ_G = 1.00 and γ_Q = 0 where loads are favourable). From this point, the incremental loading process is continued until load q_{ud} is reached, at which, based on the Eurocode definition, one region of the structure reaches the ultimate strength or there is global failure of the structure. The safety verification is then carried out evaluating the following condition:

$$E(\gamma_G G_k + \gamma_Q Q_k) \leq R\left(\frac{q_{ud}}{\gamma_{Rd}\gamma_O}\right) \tag{1}$$

where $E(X)$ is the effect of action X, $R(Y)$ represents the resistance under load Y, $\gamma_O = 1.20$ is the overall safety factor, and $\gamma_{Rd} = 1.06$ is the partial factor for model uncertainty for resistance. An example of applications of this safety format to steel-concrete composite girders is presented in (Zona et al., 2010). Being this work dedicated to the analysis of the slip demand in the shear connection, attention is given on the left-hand-side of Equation (1), with particular focus at the slip behaviour exhibited at the design load levels, i.e. when subjected to the sum of $\gamma_G G_k$ and $\gamma_Q Q_k$.

PARAMETRIC ANALYSIS

Design parameters and assumptions

The parametric study carried out involves the analysis of 1680 composite steel-concrete beams with solid slabs designed in accordance with Eurocode 4 and obtained varying the design parameters reported in Table 1. The constructional steel is of grade S355 (nominal yield strength f_y = 355 N/mm^2 and nominal ultimate strength f_u = 510 N/mm^2), while the steel for the reinforcement is of grade B450 (characteristic yield strength f_{yk} = 450 N/mm^2 and characteristic ultimate strength f_{tk} = 540 N/mm^2). Three different concrete classes are considered, i.e. characteristic compressive strengths f_{ck} = 25, 32, and 40 N/mm^2. Two variable load conditions are investigated, one for a typical office building floor with Q_k = 3.00 kN/m^2 and one for a typical garage floor with Q_k = 5.00 kN/m^2. The floor system is formed by parallel composite beams, with span lengths L varying between 10 m and 40 m, whose spacing ranges between 2.50 and 3.50 m. Given that the shape of the steel beam influences the design of the composite cross-section, three steel section shapes with double-T profiles are considered (Figure 2): sections with equal flanges (referred to in the following as sections S1), sections with the area of the bottom flange equal to twice the one of the top flange (denoted as sections S2), and sections with bottom flange area three times the top flange area (referred to as sections S3). The reinforced concrete solid slab is assumed be cast in place and three constant thicknesses are used: 125 mm (referred to as C1 in the following), 175 mm (C2), and 225 mm (C3) with 0.5% geometric reinforcement ratio. The effective slab width is taken constant and equal to the smaller value between L/4 and the spacing between two adjacent beams in accordance with Eurocode 4 Paragraph 5.4.1.2.

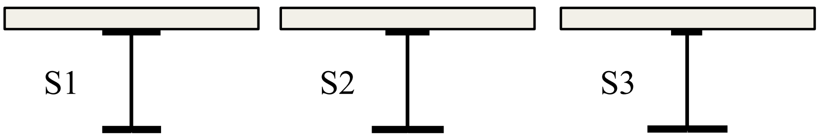


Figure 2 Cross-section shapes considered in the design.

Table 1 Considered design variables

Design parameter	Considered values
Span length L (m)	10, 15, 20, 25, 30, 35, 40
Girder spacing i (m)	2.50, 3.50
Slab thickness h_c (mm)	150, 175, 225
Variable load Q_k (kN/m^2)	3.00, 5.00
Ratio between bottom flange to top flange areas in the steel beam r	1, 2, 3
Degree of shear connection η	0.4, 0.7, 1.0
Shear connection distribution	uniform, non-uniform 2 tracts, non-uniform 3 tracts
Steel yield strength (N/mm^2)	355
Concrete compressive strength f_{ck} (N/mm^2)	25, 32, 40
Construction sequence	propped, unpropped

Three levels of shear connection are considered: one full shear connection design with connection degree 1.0 and two partial shear connection designs with connection degree 0.4 and 0.7. Three distributions of shear connection (equivalent in terms of total amount of shear connectors) are adopted for each beam and for each shear connection degree (Figure 3): uniform (U), piecewise uniform over two (N2) and three (N3) segments, where the latter two arrangements have stronger connections near the supports.

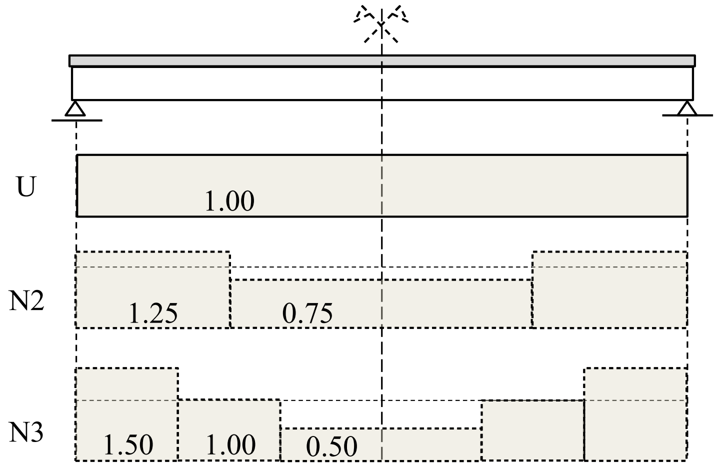


Figure 3 Shear connection distributions considered.

The construction sequences considered are propped (PR) and unpropped (UPR), resulting in different design procedures and relevant verifications. In propped construction, the design is based on the conditions: (i) the composite beam has to resist the entire permanent and variable load applied; (ii) the composite beam has to have adequate stiffness to limit vertical deflections to L/250 under service conditions (loads are G_k and Q_k for the short-term verification and G_k for the long-term verification in the hypothesis of 55% relative humidity). Given that the cross-sectional capacity is computed with plastic theory, only compact cross-sections are considered, i.e. sections of class 1 or class 2 according to Eurocode 4. In unpropped construction, the design is based on the conditions: (i) the steel beam has to carry the reinforced concrete slab when cast in place; (ii) the steel beam has to have adequate stiffness to limit deflections to L/250 during casting; (iii) the composite beams has to carry the entire subsequent load; (iv) the composite beams has to have sufficient stiffness to limit deflections to L/250 including the deflection already developed in the first construction stage. In the case of unpropped construction, both the resistance of the steel beam and the resistance of the composite beams are computed with plastic theory, thus, attention is given to verify that in both situations the cross-sections are compact (class 1 or class 2).

The designed beams are labelled with the first part indicating propped or unpropped construction (PR or UPR), followed by the indication of the shear connection degree (10, 07 and 04 for η = 1.0, 0.7, and 0.4, respectively), the shear connection distribution (U, N2, or N3), the slab thickness (C1, C2 or C3), the steel section shape (S1, S2 or S3), the design load entity (E1 and E2 for office floor with beam spacing of 2.50 and 3.50 m, respectively, E3 and E4 for garage floor with beam spacing of 2.50 and 3.50 m, respectively). The concrete strength is f_{ck} = 25 N/mm^2 unless otherwise noted.

Results

Given the space limitation for this paper, results for 84 of the 1680 beams considered in the parametric analysis are presented and discussed. For a complete report of the outcomes of the illustrated parametric study the interested readers are referred to (Zona and Ranzi, 2014).

The maximum slip demand when the designed beams reach the design load ($1.3G_k + 1.5Q_k$) is plotted in Figure 4 for the of full shear connection designs (degree of shear connection η = 1.0) with propped (left-hand side diagrams) and unpropped (right-hand side diagrams) construction sequences. Diagrams in the first row are relevant to the uniform (U) shear connection distribution, diagrams in the second and third rows to the non-uniform N2 and N3 shear connection distributions, respectively. Each single diagram reports the maximum slip demand calculated for 84 composite beams: 12 beams for a given span length having three steel cross-section shapes (S1, S2, and S3) and four design load conditions (E1, E2, E3, and E4). Three different marks are used to indicated the slip demand for the three cross-section shapes, i.e., circle for S1, x-cross for S2, and triangle for S3. The load conditions are not marked in the diagrams in order to avoid too many symbols, nevertheless, the four design loads can be identified knowing that the slip demand increases moving from E1 to E4.

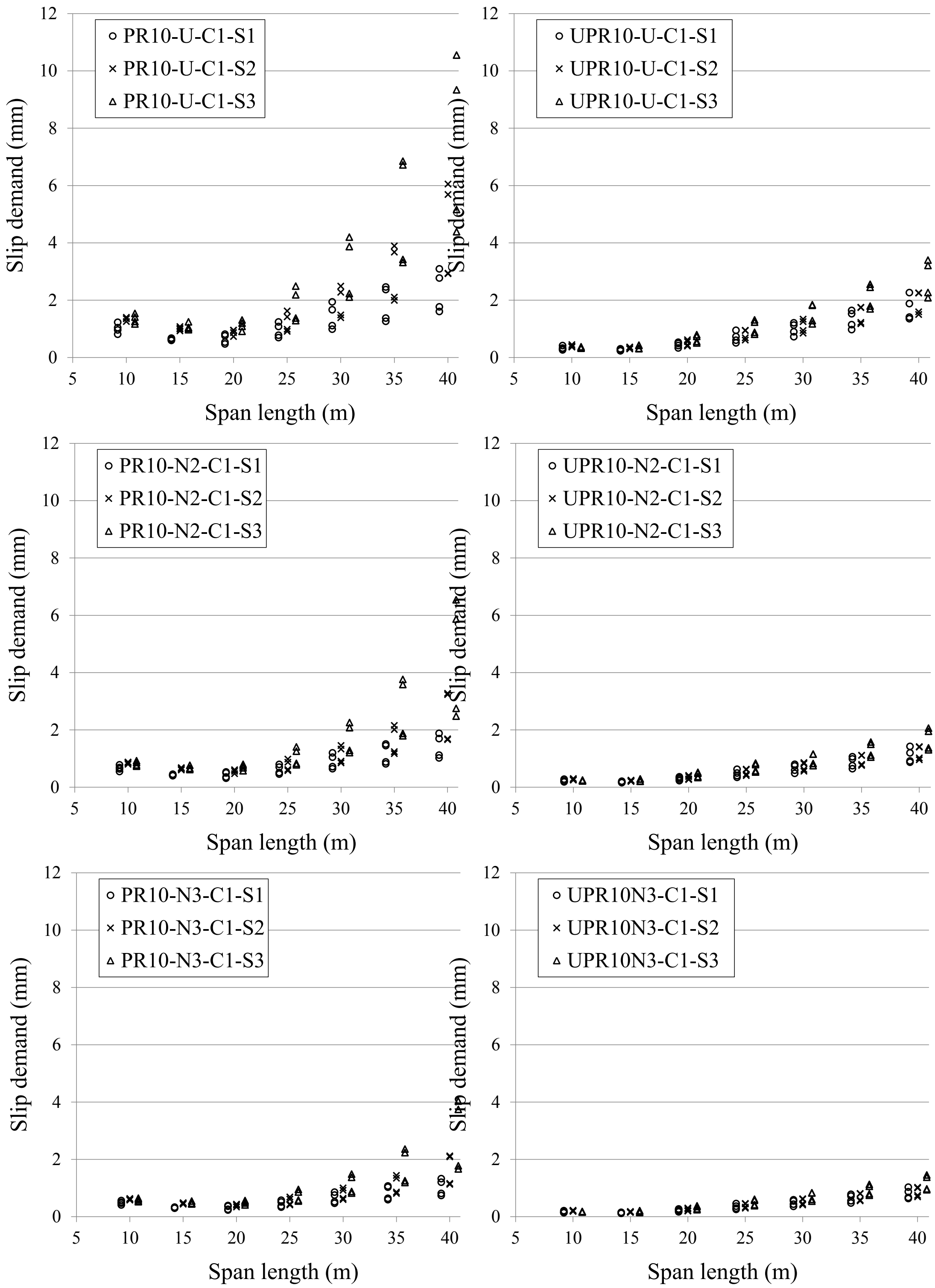


Figure 4 Slip demand at design load for propped (left-hand side) and unpropped (right-hand side) beams with full shear connection.

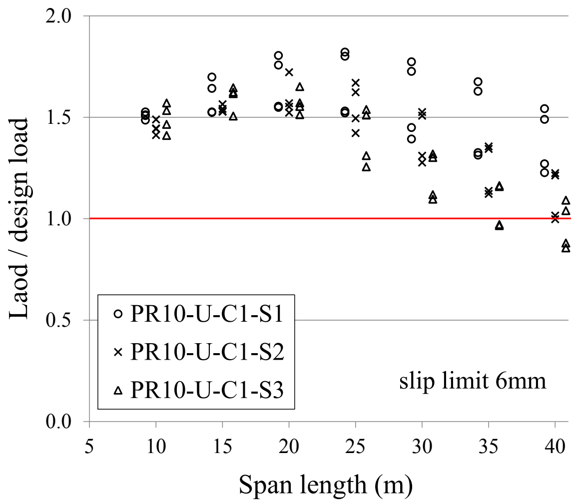
It is observed that: (i) the slip demand steadily increases with the span for lengths larger than 20 m; (ii) the ratio between the top and bottom flange of the steel cross-section has a significant influence on the slip demand (slip increases from shape S1 to shape S2 and further to S3) and this effect is more important for longer spans, particularly for propped construction; (iii) the construction sequence has a major influence on the slip demand with the propped design (PR) demanding slips about three times the slips in the corresponding unpropped design (UPR) cases; (iv) the shear connection non-uniform distributions (N2 and even more N3) have a substantial effect on the reduction of the maximum slip when compared to the uniform distribution (U).

The first three observations can be explained considering that in composite beams the slip demand increases: (a) when the distance between the shear connection and the plastic neutral axis of the composite cross-section grows and all other parameters are fixed; (b) when the span-to depth ratio of the composite beam is incremented and all other parameters are fixed. When the span length grows, the plastic neutral axis lowers its position given that the thickness of the concrete slab is kept constant while the steel beam necessarily increases its cross-section. Hence, a longer span length has a larger distance between the shear connection and the neutral axis, that in turn causes the observed increment in the slip demand. The effect of the shape of the steel section has a similar interpretation, as a non-symmetric cross-section with bigger bottom flange allows a higher span-to-depth ratio (causing increased slip demand) and simultaneously lowers the neutral axis (also causing increased slip demand).

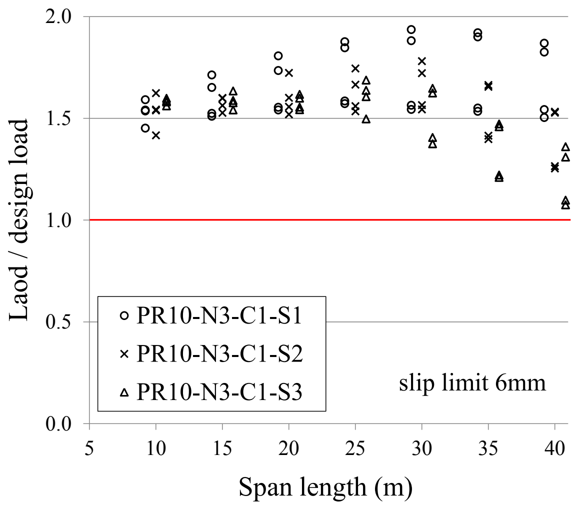
Regarding the differences in slip demand observed between propped and unpropped construction, they are inherent to the design assumptions. Given that, in the unpropped construction, the steel beam has to resist alone the applied gravitational loads during slab casting, a stronger and stiffer steel beam is adopted, resulting in a smaller span-to-depth ratio (causing reduced slip demand) that has more important effects than the lower neutral axis (causing higher slip demand). The differences in slip demand between propped and unpropped construction are also incremented, although with a minor contribution, by the fact that, in propped construction, the beam self-weight is carried by the composite section, therefore producing slip, while this is not the case in unpropped construction, where slip starts after the concrete slab has hardened, thus, the composite beam self-weight does not contribute to the attained slip.

The variation in slip demand for the different shear connections distributions (U, N2, N3) can be explained given that the amount of required shear force redistribution along the beam span is reduced when the arrangement of the shear connectors approaches the shear distribution proportional to the shear force, as can be seen, for example, from the shear force distributions produced by arrangements U, N2 and N3.

The influence of the magnitude of the variable load assumed in the design appears generally small but not negligible for beams with full shear connection and span lengths less than 25 m. Slightly higher slip demands are required with the garage floor loads (E4 and E3) when compared to the cases subjected to office building floor loads (E2 and E1). The differences between solutions having the same span length, but designed with different variable loads, become more pronounced for span lengths equal or larger than 25 m and for lower degrees of the shear connection, with constantly higher slip demands for higher variable loads, i.e. moving from E1 to E4. This effect can be explained as a higher design load, when all other design parameters are fixed (i.e. concrete slab, steel beam shape, materials, shear connection degree, construction sequence), results in generally stronger steel cross-sections, obtained in the parametric study by increasing the top and bottom flange areas while keeping their ratio constant as well as maintaining the overall steel section height unmodified. Such an increase in flange area leads to a lowering of the neutral axis which in turn causes the increase in slip demand.
The results presented in Figure 4 show the slip demand at the design load ($1.3G_k + 1.5Q_k$) under the assumption of unlimited slip capacity (infinite ductility) of the shear connection. In Figure 5 the magnitude of the load obtained limiting the maximum slip to 6 mm to highlight the effect of the actually limited shear connection slip capacity on the overall load carrying capacity of composite beams. If the maximum slip attained during the nonlinear analysis is below the assigned limit, then the beam load carrying capacity is not affected by the shear connection limitation. On the other hand, if the slip reaches the assigned limit value, the nonlinear analysis is terminated and the corresponding load level is taken as the carrying capacity for the composite beam. Results are given for the case of propped construction shown to be the most critical cases in terms of slip demand. It is observed that for spans longer that 30 m the beams with section S3 and uniform shear connection distribution do not reach the design load when the shear connection slip is limited to 6 mm. As already noted, more results and details can be found in reference (Zona and Ranzi, 2014).



(a) slip limited to 6 mm



(b) slip limited to 8 mm

Figure 5 Maximum carried load for shear connection slip limited to 6 mm and 8 mm for propped beams with full shear connection.

CONCLUSIONS

In this work an extensive parametric study involving simply-supported composite steel-concrete beams with solid slabs designed according to Eurocode 4 was conducted to evaluate the influence of various parameters on the predicted slip demand in the shear connection at the ultimate limit state. The presented results show that, for a given shear connection degree, three design parameters are the most influential on the slip demand in the shear connection: construction sequence (propped construction results in slip demand always larger than unpropped construction); span length (longer spans are the most critical in terms of slip demand); steel section shape (sections with equal flanges are less critical). In addition, the shear connection distribution can have an important effect as the non-uniform shear connection distributions with more connectors near the supports are effective in limiting the slip demand mostly for the full shear connection designs. While the span length and the steel section shape are considered in current European design codes, no mention is explicitly made to the construction sequence and to the distribution of shear connectors.

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REFERENCES

European Committee for Standardization (2004) *Eurocode 4: Design of composite steel and concrete structures - Part 1-1: General rules and rules for buildings*. EN 1994-1-1.

Oehlers, D.J., Nguyen, N.T., Ahmed, M. and Bradford M.A. (1997) “Partial interaction in composite steel and concrete beams with full shear connection”, *Journal of Constructional Steel Research*, 41(2/3), 235-248.

Oehlers, D.J. and Sved, G. (1995) Composite beams with limited-slip-capacity shear connectors, *Journal of Structural Engineering*, 121(6), 932-938.

Ollgaard, J.G., Slutter, R.G. and Fisher, J.W. (1971) “Shear strength of stud connectors in lightweight and normal weight concrete”, *AISC Engineering Journal*, 2(1), 55-64.

Zona, A., Barbato, M., Dall’Asta, A. and Dezi, L. (2010) “Probabilistic analysis for design assessment of continuous steel-concrete composite girders”, *Journal of Constructional Steel Research*, 66(7), 897-905.

Zona, A. and Ranzi, G. (2014) “Shear connection slip demand in composite steel–concrete beams with solid slabs”, *Journal of Constructional Steel Research*, 102(1), 266-281.

Zona, A. and Ranzi, G. (2011) “Finite element models for nonlinear analysis of steel-concrete composite beams with partial interaction in combined bending and shear”, *Finite Elements in Analysis and Design*, 47(2), 98-118.